Appendix F
Effect of Vertical Shear on the Stability of Gravity Walls

F-1. Introduction

This appendix provides a simplified design procedure to calculate the vertical shear (downdrag) force on gravity walls due to backfill settlement. The downdrag force represents the total shear force on a vertical plane extending through the backfill from the heel of the wall. The downdrag force improves stability and therefore should be considered in those cases where the wall does not meet stability requirements if the downdrag is ignored. Downdrag may be mobilized either due to the movement of the wall as the backfill is placed, or due to settlement of the backfill; however, this appendix is limited to the case of non-moving walls with downdrag forces induced by backfill settlement. Backfill settlement introduces shear strains in the backfill material, which produce the vertical downdrag force. Once the horizontal force (F_H) has been determined and once the downdrag force (F_V) is computed using the procedures described in this appendix**,** the mobilized friction angle (δ) along a vertical plane extending through the backfill from the heel of the wall can be determined as $\delta = \tan^{-1} F_V/F_H$. It is generally not necessary to determine the value of δ; however, this may be useful if general wedge solutions are to be applied subsequent to determining the value of F_V.

F-2. Background

a. Previous design practice. In most cases of massive retaining walls constructed on rock foundations, movements of the wall and backfill are not sufficient to fully mobilize the shear resistance of the soil. Past practice has been to determine lateral earth pressures acting against the back of the gravity wall and set the interface friction between the wall and the backfill equal to zero. Zero interface friction along the back of the wall corresponds to a zero shear force along the back of the wall.

b. Finite element analysis. To develop an improved understanding of the interaction between gravity walls, their foundations, and their backfills, an investigation using finite-element analyses was conducted (Ebeling et al. 1990 and 1992; Ebeling, Duncan, and Clough 1990). The analyses demonstrated that the backfill settles relative to the wall and develops downward-shear loads on the wall. The magnitude of the vertical shear force (F_V) is expressed in terms of a vertical shear coefficient (K). This coefficient is related to the shear force on the vertical plane through the heel of a wall by the following equation (note that some of the nomenclature used in this appendix has been altered from the nomenclature used in referenced source material):

$$F_V = K \left(\frac{\gamma \ H^2}{2} \right) \tag{F-1}$$

where: γ = unit weight of backfill
H = depth of backfill against wall

Figure F-1 shows the results of finite-element analyses of four walls founded on rock and retaining a moist backfill, where values of K range from 0.09 to 0.21. The analyses indicated that rock founded gravity walls would move only slightly during the placement of the toe fills and backfills. As a result, the earth pressures on the backs and fronts of the walls are close to those that exist at rest. Even so, settlement of the backfill as it is placed behind the wall is sufficient to generate a significant amount of shear force. Parametric studies demonstrated that the most important factors influencing the value of K for concrete gravity walls on rock foundations are the depth of the backfill, the stiffness of the backfill, the inclination of the back of the wall, and the number of steps in the back of the wall. The following trends were observed:

- For low walls, the value of K increases with increasing wall height because more backfill compression occurs due to self-weight of the backfill. The resulting increase in differential movement between backfill and the wall causes a greater portion of the interface strength to be mobilized. This process approaches a limiting condition for high walls as the interface strength becomes fully mobilized over most of the wall-backfill contact area.

- As the stiffness of the backfill increases, backfill compression decreases, and the wall height necessary to mobilize the full interface strength increases. For low walls with vertical back sides, the value of K decreases as the backfill stiffness increases.

- The value of K decreases as the back side of the wall becomes inclined away from the backfill.

- The value of K is greater for a wall with a stepped-back side than for a wall with a smooth-back side at the same average slope.

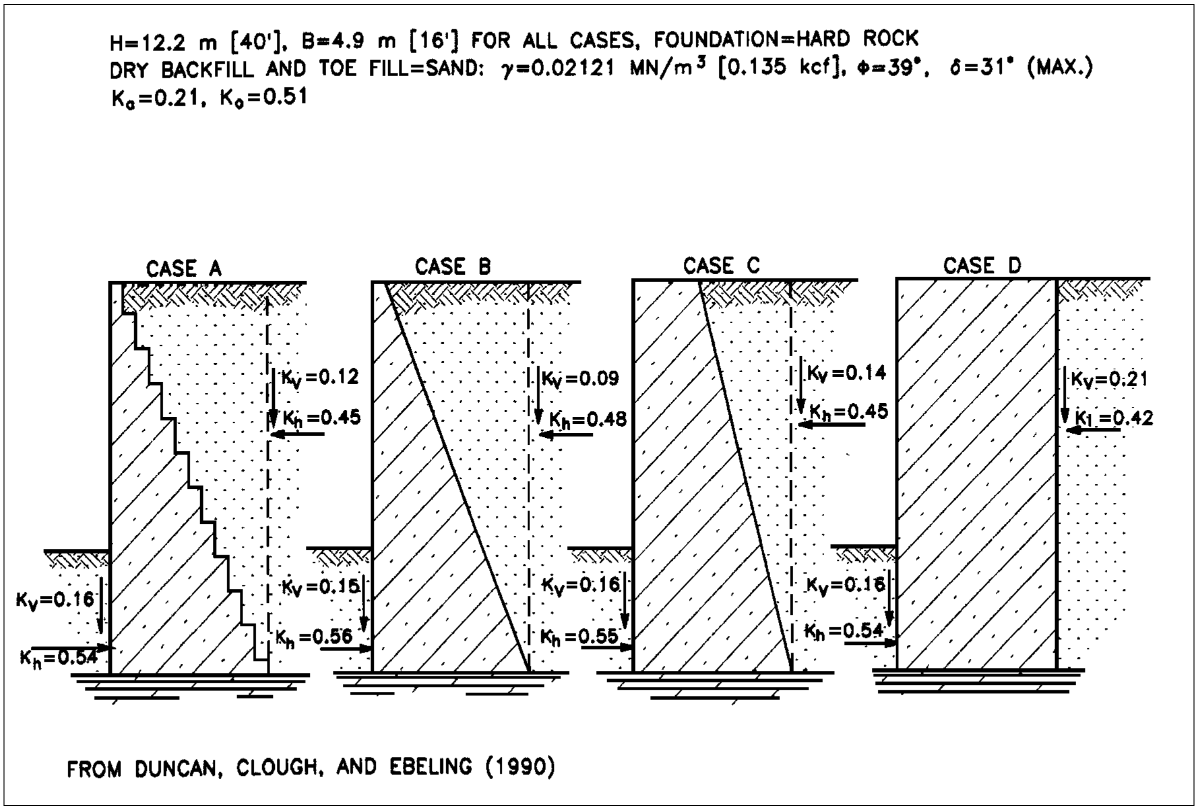


Figure F-1. Results of finite element analysis of four walls founded on rock-retaining dry backfill (K = value of K_V shown in this figure)

c. Field and model wall measurements. Shear loads have been reported for several instrumented walls (Duncan, Clough, and Ebeling 1990 and Filz, Duncan, and Ebeling 1997), including a lock wall 30.2 m (99.1 ft) in height and founded on rock (Hilmer 1986). Measurements at the lock wall are reported over a 6-year period. Mobilized interface friction at the lock wall fluctuates seasonally and with changes in the water level inside the lock. However, the data indicate that the shear force is persistent over the 6-year period and does not decay with time. According to a conservative interpretation of the data, the minimum value of K during the 6-year period is about 0.18. In a research program conducted at Virginia Polytechnic Institute and State University (Filz and Duncan 1992), both the horizontal earth force and the vertical shear force along the vertical back side of a 2.1 m (7 ft) high rigid retaining wall were measured. The research program included 16 tests using compacted fine sand (Unified Soil Classification SP) and compacted non-plastic silty sand (SM) as backfill. Measured values of K ranged from 0.11 to 0.23 (Table 8.9 in Filz and Duncan 1992 or Table 6.2 in Filz, Duncan, and Ebeling 1997). The more compressible backfills exhibited higher K values. The compacted backfills were left in place for periods ranging from 1 to 14 days after completion of backfilling. Values of K tended to increase with time.

d. Benefits. Filz, Duncan, and Ebeling (1997) present an example calculation using vertical shear for a 30-ft-high, step-tapered, rock-founded, gravity wall retaining dense sand with surcharge (no ground water table). This example compares the result with a conventional design and shows a 14 percent reduction in base width by including vertical shear, without compromising the design safety requirements.

e. Limits on use of vertical shear. The vertical shear force can be incorporated in conventional equilibrium calculations, subject to the following limitations:

- When a toe fill of significant height exists, a vertical shear force at the toe should be included in the equilibrium calculations if a vertical shear force was applied to the back of the wall. Neglecting the shear force at the toe could result in unconservative estimates of the base contact area and the maximum bearing pressure on the foundation.

- The vertical displacements within the foundation during construction of the wall and placement of the backfill are negligible when compared with the vertical settlement within the backfill due to self-weight. Gravity walls founded on competent rock foundations satisfy this criterion.

- The backfill does not creep. Compacted soils classified as SW, SP, GW, and GP according to the Unified Soil Classification System (ASTM 1990) do not experience significant creep movements, nor do select SM backfills with non-plastic fines that do not creep.

- No special features that reduce or eliminate interface friction exist along the interface between the back of the wall and the backfill. Examples of special features that would reduce interface friction include bituminous coatings and synthetic barriers with low interface friction values.

- The interface between the back side of the wall and the backfill is capable of developing friction values of $\delta > 0.7\,\varphi$, where φ is the effective angle of internal friction for the backfill. This is satisfied by SW, SP, GW, and GP soils compacted against concrete walls. It is also satisfied by SM soils with non-plastic fines compacted against concrete walls.

- The water pressures are hydrostatic within the backfill, and the rise in water table is concurrent with placement of the soil lifts.

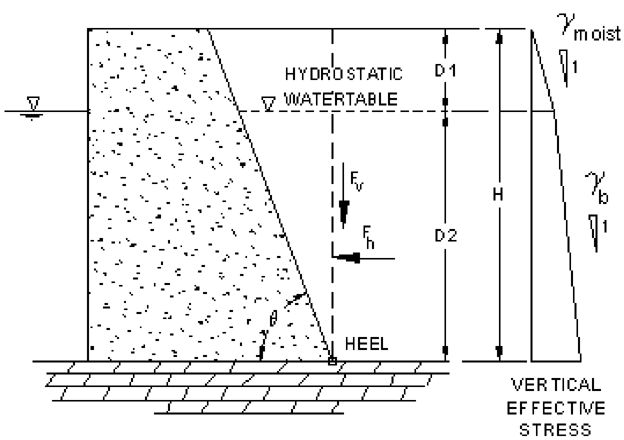
F-3. Procedures for Calculating the Vertical Shear Force

Two basic procedures for computing the magnitudes of shear loads along the backs of gravity walls are described in this section: a simplified procedure and a complete soil-structure interaction analysis using finite elements. These procedures are intended only as guidelines and are not intended to replace judgment by the project engineers.

a. Simplified procedure for level backfill, with no surcharge. For level backfill, with no permanent surcharge, the downdrag on a vertical plane through the heel of the wall can be computed as:

$$F_v = K\left[\frac{1}{2}\gamma_{moist}(D_1)^2 + \gamma_{moist}(D_1 D_2) + \frac{1}{2}\gamma_b(D_2)^2\right] \qquad \text{(F-2)}$$

where: K = the vertical earth force coefficient
D_1 = thickness of backfill above the hydrostatic water table
D_2 = thickness of submerged backfill above the base of the wall
$H = D_1 + D_2$ = total height of backfill
γ_{moist} = moist unit weight of backfill (above the water table)
γ_{sat} = saturated unit weight of submerged backfill
γ_w = unit weight of water
γ_b = buoyant unit weight of submerged backfill = $\gamma_{sat} - \gamma_w$



This equation assumes that the water pressures are hydrostatic within the backfill and the rise in water table is concurrent with the placement of soil lifts. The term in the bracket represents the integral of the vertical effective overburden stress shown in Figure F-2. (The case of walls in which there is a post-construction rise in groundwater level in the backfill will be discussed later in this appendix). For the case of dry backfill, D_2 equals zero and Equation F-2 reduces to Equation F-1. In Equation F-2, the value of K is obtained from Equation F-3:

$$K = (1 - C_\theta C_N) K_V \tag{F-3}$$

Figure F-2 Forces acting on vertical plane at heel of wall

where: K_V = from Figure F-3,
C_N = correction factor for the number of steps in the backfill side, from Figure F-4,
C_θ = correction factor for inclination of the backfill side, from Figure F-4,

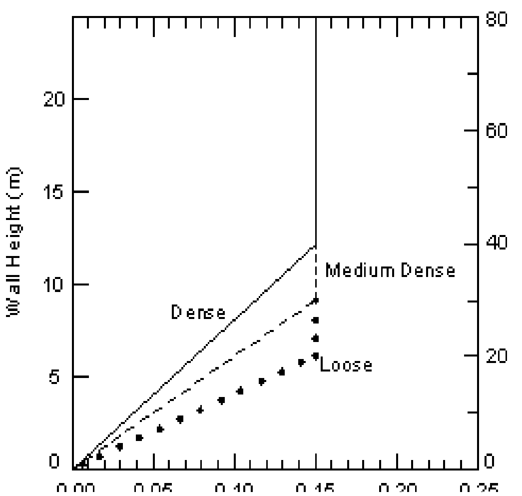


Figure F-3 Design values of K_v for gravity walls founded on rock, with a vertical face against the backfill

Figure F-3 shows that the value of K_V increases with increasing wall height until a limiting value of 0.15 is reached, and that the limiting value develops at lower heights for walls with loose backfill than for walls with dense backfill. The recommended design value is well below the actual value indicated by measurements and analyses. It was selected to be conservative because it is a simplified design procedure. Even with this conservative selection of the design value, significant economies can be obtained by including the vertical shear force in design.

b. Simplified procedure for surcharges. In the case of rock-founded gravity walls with surcharges on the backfill, F_v is calculated using the following procedure. This procedure can also be used to simulate the effects of sloping backfill surfaces, where the additional weight of backfill above some horizontal plane can be represented as an idealized surcharge. Terminology is the same as used for flat backfills without surcharges, unless otherwise noted.

$$F_v = K \left[\tfrac{1}{2} \gamma_{moist} (D_1)^2 + \gamma_{moist} (D_1 D_2) + \tfrac{1}{2} \gamma_b (D_2)^2 \right] + K' \; C_s (q) H \tag{F-4}$$

where all variables are identical to equation F-2, except:

$K' = 1.4\,K$, but $K' \leq 0.15$

C_S = the correction factor for an inclined backfill surface, from Figure F-3,

q = surcharge pressure on the backfill (for sloping backfills $q = \gamma_{moist}\ \Delta H$).

The reason for use of K' in the last term of this equation, rather than K, is that the surcharge load increases the backfill settlement, thus resulting in greater vertical shears. This procedure should only be used for permanent surcharges; temporary surcharges might not be sufficient to significantly increase backfill settlement.

c. Post-construction rise in ground water. Equations F-2 and F-4 assume that the rise in water table within the backfill is concurrent with placement of the soil lifts. In many cases, the backfill is placed before submergence takes place. For walls in which the construction of the wall and the placement of "dry" (or more precisely, moist) backfill precedes a rise in the ground water level, a soil-structure interaction analysis procedure is generally used to compute the values of F_v for both a dry backfill condition and a post-construction rise in ground water condition. In these situations, a rebound of the soil can occur. This results in a reduction in the effective stress and can result in a reduction in the shear force Fv. This occurred in analysis of the soil-founded U-frame lock at Red River Lock No. 1 (Ebeling et al. 1993, Ebeling and Mosher 1996). The soil-structure-foundation interaction analysis of the new roller-compacted concrete (RCC) lock (rock founded) at McAlpine Locks by Ebeling and Wahl (1997) is another example of this type of analysis. Both structures were constructed in the dry, followed by partial submergence of the lock sites. However, a simplified procedure is also available to compute values for F_v after backfilling and after a post-construction rise in ground water level. This procedure is based on data from the soil-structure-foundation interaction analysis of the new roller-compacted lock wall at McAlpine Locks (Ebeling and Wahl 1997). Due to limited data, this procedure is restricted to level backfills with no surcharge. Two values for F_v are computed for the two stages of wall construction. The first stage being that which occurs after backfilling is complete. The second stage being that which occurs after a post-construction rise in the ground water level.

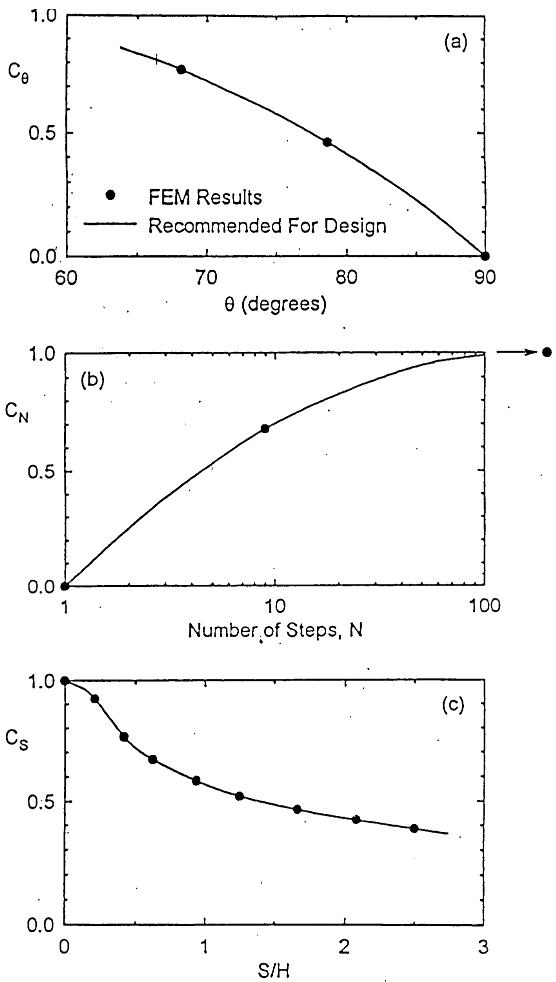
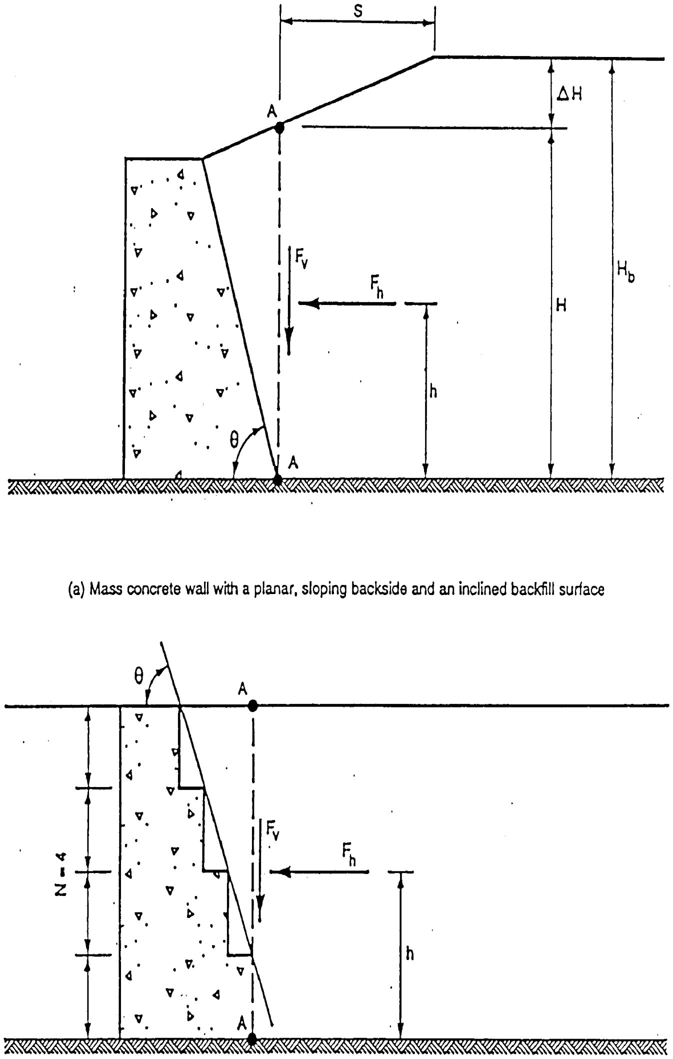


Figure F-4 Correction factors C_θ, C_N, and C_S (geometry definitions are illustrated in Figure F-5)

(1) Stage 1. F_v after backfilling is computed using Equation F-1, or Equation F-2 with D_2 set equal to zero.

(2) Stage 2. After the post-construction rise in the ground water level, F_v is calculated using equation F-2, but substituting $K\,C_{wt}$ in place of K. The value for the correction factor C_{wt} is given by:

$$C_{wt} = \left\{1 - \frac{D_2}{H}\right\} \tag{F-5}$$



(a) Mass concrete wall with a planar, sloping backside and an inclined backfill surface

(b) Mass concrete wall with a stepped backside

Figure F-5 Geometry definition

d. Soil-structure interaction analysis. A complete soil-structure interaction (SSI) analysis is recommended for those structures for which the simplified procedure is not applicable, or for those cases in which a more precise evaluation of the shear force is required. Soil-structure analyses are recommended for U-frame locks, retaining structures founded on soils, and structures with complicated geometry. This can be accomplished using a finite element program such as SOILSTRUCT (Ebeling, Peters, and Clough 1992). Unlike conventional equilibrium procedures, an SSI analysis does not require the use of predetermined pressure distributions between the soil and the wall. Instead, it allows for development of these pressures through soil-structure interaction by simulating the staged construction that occurs. The computer program SOILSTRUCT can model the nonlinear stress-strain behavior of the soil and allow for relative movement between the soil and the structure by incorporating interface elements in the mesh. SSI analyses are also especially useful for analyzing retaining structures founded on either soils or compressible rock foundations. Differential settlements within the foundation affect the magnitude of the shear force that the backfill exerts on the wall. The SSI analysis procedure has been successfully used for a wide variety of problems, including the Port Allen and Old River locks (Clough and Duncan 1969) and, more recently, the lock at Red River Lock and Dam No. 1 (Ebeling et al. 1993, Ebeling and Mosher 1996, and Ebeling, Peters, and Mosher 1997). Additional information on soil-structure Interaction analysis can be found in Ebeling, Pace, Morrison (1997).